88

VOLUME 81 SEPARATE No. 588

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

JANUARY, 1955

CONTINUOUS PRESTRESSING

by Robert B.B. Moorman, M. ASCE

STRUCTURAL DIVISION

{Discussion open until May 1, 1955}

Copyright 1955 by the AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

THIS PAPER

--represents an effort by the Society to deliver technical data direct from the author to the reader with the greatest possible speed. To this end, it has had none of the usual editing required in more formal publication procedures.

Readers are invited to submit discussion applying to current papers. For this paper the final date on which a discussion should reach the Manager of Technical Publications appears on the front cover.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying "Publication Procedure for Technical Papers" (Proceedings — Separate No. 290). For free copies of this Separate—describing style, content, and format—address the Manager, Technical Publications, ASCE.

Reprints from this publication may be made on condition that the full title of paper, name of author, page reference, and date of publication by the Society are given.

The Society is not responsible for any statement made or opinion expressed in its publications.

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

CONTINUOUS PRESTRESSING

Robert B. B. Moorman,[1] M. ASCE

SYNOPSIS

The nature and treatment of prestress loads for continuous prestressed concrete beams are discussed. Two examples are worked out. An approximate method of determining friction losses is discussed. A procedure for analysis and design is also included.

INTRODUCTION

Continuous beams whether prestressed or not are subjected to moments due to continuity. For non-prestressed beams the moments are caused by dead load and applied loads. For prestressed beams the same dead load and applied load moments occur. However, in addition to these, there are moments due to the effects of prestressing.

The effects of prestressing can be expressed as lateral loads[(1)(2)(3)(4)(7)(8)(9)] and longitudinal loads, either axial or eccentric. Any of the usual methods can be used to analyze the concrete structure to determine the effects of the prestressing loads.

If the principle of superposition is accepted as valid in the case of prestressed statically indeterminate structures, the dead load, applied loads, and prestressing forces may be dealt with separately. The resulting moments may be added algebraically to obtain the actual maximum and minimum values for the loaded structure.

Assumptions

In this paper it is assumed:
that Hooke's Law is valid,
that the principle of superposition holds,
that the horizontal component is equal to the tension in the cable,
that the friction force is negligible,
that the lateral force from the cable is vertical, or, normal to the axis of the member,
that the loss of cable tension caused by creep is negligible,
that the reduction in cross-sectional area due to cable ducts is negligible.

Nature of Prestress Loads

The prestressing unit produces loads on the concrete in the form of reactions at the end anchorages, lateral reactions between the cable and sheath (for curved cables), tangential frictional reaction along the cable, and external reactions caused by bending deformation of the beam when the beam is

1. Chairman, Dept. of Civ. Eng., Syracuse Univ., Syracuse, N. Y.

statically indeterminate. Figure 1 shows the horizontal and vertical end anchorage forces, the lateral force, and the frictional force acting on a portion of a beam.

For purposes of analysis, prestressed concrete may be considered a homogeneous material. Then it only remains to take into account the forces applied to the concrete structure. These forces may consist of end anchorage reactions, normal reaction along cable, frictional reaction along cable, dead load, various types of live loads, and the reactions produced by the foregoing loads.

It cannot be overemphasized that in the initial stages of analysis of a prestressed concrete beam, it should be treated as a member consisting of a homogeneous material. The beam should be analyzed for the usual live and dead loads. It then should be analyzed for the prestress loads. The principle of superposition may be used to find the net results of the analyses.

In the computation of the ultimate or failure loads on the beam it is considered to act as a reinforced concrete beam. In such a case, an ultimate theory should be used in the stress and moment analysis.

Equivalent Load

If M is the bending moment caused by an eccentric cable, T the resultant (horizontal) tension in the cable, and e the eccentricity to the resultant tension,

$$M = -Te$$

If T is assumed constant and if the expression $\frac{d^2M}{dx^2} = -w$, from the study of mechanics of materials, is used,

$$\frac{dM}{dx} = -T\frac{de}{dx}$$

$$\frac{d^2M}{dx^2} = -T\frac{d^2e}{dx^2} = -w_p$$

in which w_p is an equivalent load resulting from an eccentric prestressing cable.

Sometimes it is not possible to express e in a convenient mathematical form. In such instances the following expression is sufficiently accurate for all practical purposes

$$\frac{d^2e}{dx^2} = \frac{e_1 - 2e_2 + e_3}{(\Delta x)^2} \quad \text{approximately}$$

in which e_1, e_2, and e_3 equal the eccentricities at any three consecutive points along the beam axis and Δx the distance between the points measured along the beam. The equivalent load then may be expressed as

$$w_p = T\left(\frac{e_1 - 2e_2 + e_3}{(\Delta x)^2}\right)$$

This expression may be used for members of variable section as well as of constant section.

It should be noted that e is positive when the distance is below the centroidal axis, that when w_p is negative the load acts upward, and that the beam is divided into equal length segments Δx. This is shown in Fig. 2.

If the post-tensioning cable is straight between supports, the equivalent load w_p is used only for determining the bending moments after which it is set equal to zero for computing the shears and reactions. This is shown in Fig. 3.

If the member is straight and the post-tensioned cable is curved, the equivalent load w_p should be used for determining bending moments, shears, and reactions.

If the equivalent load w_p is based on an e that results from curvature of the post-tensioned cable as well as curvature of the axis of the beam, there are two loads to consider in the analysis. The total w_p is used to compute the bending moments. Then, to find the portion of w_p to be employed in the calculations for reactions and shears, select any convenient horizontal straight line, above the cable, as a datum to determine the e values to be used in the equivalent load expression.

When the cable is placed as shown in Fig. 4 the equivalent load is concentrated. The expression for it is

$$P_p = -T(\tan \alpha + \tan \beta)$$

When a parabolic shaped cable has a span of L and a sag e the equivalent load is

$$w_p = -\frac{8Te}{L^2}$$

When the end anchorage is eccentric as in Fig. 5, the equivalent loading consists of moments at the ends as shown.

Parabolic Curves

Probably the most suitable cable form is parabolic. At least it is the simplest form with which to work.

Figure 6 shows a continuous curve consisting of three parabolas. The equations for the curves are shown thereon. In order that the two curves on the right be tangent at their common point the following conditions must be satisfied

$$C_1 = 2a_1C \quad \text{and} \quad C_2 = 2a_2C$$

Reading from left to right the equations for the equivalent loads caused by curvature are

$$w_p = -\frac{8TC_A}{L^2}$$

$$w_p = -\frac{4TC}{a_1L^2}$$

$$w_p = + \frac{4TC}{a_2 L^2}$$

respectively.

For preliminary design purposes, the shape in Fig. 7 may be used for end spans and the shape in Fig. 8 for intermediate spans. The initial values of a may be taken as $a_1 = 0.35L$ and $a_2 = 0.15L$. After an analysis, the designer can adjust these values in any manner he deems desirable.

Examples

Say the section shown in Fig. 9 is to be used for a two-span continuous beam. Each span is to be 70 ft. In addition to the weight of the beam, 653 lb per ft, there is a live load of 1066 lb per ft which may be placed in either or both spans. A working stress of 2000 psi in the concrete is to be used.

The maximum allowable cable tension may be computed as

$$T = (2000)(627)\left(\frac{27.78}{57}\right) = 610{,}000 \text{ lb, say}$$

The maximum dead and live load moments which occur at the center support may be computed as

$$M\ (D.L.) = -(1/8)(653)(70)^2 = -400{,}000 \text{ ft-lb} = -4800 \text{ in.-kips}$$

$$M\ (L.L.) = -(1/8)(1066)(70)^2 = -652{,}900 \text{ ft-lb} = -7834.8 \text{ in.-kips}$$

In the following examples two shapes for the cables are used, namely, (1) the single parabolic shape as in Fig. 10 and (2) the shape consisting of five parabolas as in Fig. 12.

Example 1. In this example the single parabola is used for the shape of the cable. If at the center support the eccentricity of the cable is taken as 24.5 in. downward, the equivalent load may be written as

$$w_p = -\frac{2Te}{L^2} \quad \text{(approximately)}$$

$$= -\frac{(2)(610)(24.5)}{(12)(70)^2} = -0.508 \text{ kips per ft}$$

This load is shown in Fig. 10.

The bending moment at the support caused by prestressing is

$$M_p = (1/8)(508)(70)^2 = 311{,}000 \text{ ft-lb} = 3733.8 \text{ in.-kips}$$

The curves of maximum moments are shown in Fig. 11.

The unit stresses at top and bottom of the beam in the span are

$$f_t = \frac{610}{627} + \frac{(6551.6)(27.78)}{254{,}746} = 1.687 \text{ ksi}$$

$$f_b = \frac{610}{627} - \frac{(6551.6)(29.22)}{254{,}746} = 0.221 \text{ ksi}$$

and at the support they are

$$f_t = \frac{610}{627} - \frac{(8901.0)(27.78)}{254,746} = 0.003 \text{ ksi}$$

$$f_b = \frac{610}{627} + \frac{(8901.0)(29.22)}{254,746} = 1.993 \text{ ksi}$$

Example 2. In this example the cable shape consisting of five parabolas, as in Fig. 12, is used. Let $a_1 = 0.35$ and $a_2 = 0.15$.

Using only the live and dead loads, maximum bending moments of +8642 in.-kips in the span and -12640 in.-kips at the center support are computed. The eccentricities for the prestressing cables are then computed as

$$\frac{8642}{610} = 14.167 \text{ in.} = 1.181 \text{ ft, downward at mid-span,}$$

and

$$\frac{12640}{610} = 20.721 \text{ in.} = 1.727 \text{ ft, upward at the center support.}$$

Thus, $C_A = 1.181$ ft, $C = 2.908$ ft, $C_1 = (0.7)(2.908) = 2.036$ ft, and $C_2 = (0.3)(2.908) = 0.872$ ft. The equivalent loads are

$$w_p = -\frac{(8)(610)(1.181)}{(70)^2} = -1.176 \text{ kips per ft}$$

$$w_p = -\frac{(4)(610)(2.908)}{(0.35)(70)^2} = -4.137 \text{ kips per ft}$$

$$w_p = +\frac{(4)(610)(2.908)}{(0.15)(70)^2} = +9.654 \text{ kips per ft}$$

These loads are shown in Fig. 12.

Using the coefficients shown in Fig. 13, the fixed end moments, caused by the prestressing loads, are computed at the exterior support as

$$1.176(70)^2(0.05729) = 330.13 \text{ ft-kips}$$

$$4.137(70)^2(0.02504) = 507.59$$

$$-9.654(70)^2(0.000998) = -47.21$$

$$+790.51 \text{ ft-kips}$$

and at the interior support as

$$1.176(70)^2(0.02604) = 150.05 \text{ ft-kips}$$

$$4.137(70)^2(0.04817) = 976.47$$

$$-9.654(70)^2(0.00913) = -431.89$$

$$+694.63 \text{ ft-kips}$$

The bending moment at the interior support caused by the prestressing loads is found by balancing the moment at the exterior support and carrying over -0.5 to the interior support. Thus -0.5 (-790.51) + 694.63 = 1089.89 ft-kips = 13,078.7 in-kips is the bending moment at the interior support.

The curves of maximum moments are shown in Fig. 14. The computed points through which the curves are drawn are shown as circles. It should be noted that the most favorable condition is when the beam is loaded with prestress, dead load, and live load on both spans.

The unit stresses at top and bottom of the beam for prestress plus dead load are (in the span)

$$f_t = \frac{610}{627} - \frac{(5436.3)(27.78)}{254,746} = 0.382 \text{ ksi}$$

$$f_b = \frac{610}{627} + \frac{(5436.3)(29.22)}{254,746} = 1.596 \text{ ksi}$$

and (at the support)

$$f_t = \frac{610}{627} + \frac{(8908.7)(27.78)}{254,746} = 1.944 \text{ ksi}$$

$$f_b = \frac{610}{627} - \frac{(8908.7)(29.22)}{254,746} = -0.048 \text{ ksi tension}$$

For prestress plus dead load plus one span loaded with live load, the unit stresses in the span are

$$f_t = \frac{610}{627} - \frac{(7786.7)(27.78)}{254,746} = 0.124 \text{ ksi}$$

$$f_b = \frac{610}{627} + \frac{(7786.7)(29.22)}{254,746} = 1.865 \text{ ksi}$$

The position of the resultant of the prestressing unit may be determined either by use of the equations in Fig. 6, or, by removing the center support, computing the moments caused by the prestressing loads acting on the simple beam, and then dividing this moment by T.

Friction

Friction may result from three causes: (1) from intentional passage of cable around a curve, in which case it is proportional to the subtended angle, (2) from unintentional deviation or "wobble" of duct from desired line, in which case it is proportional to the length of the duct, and (3) from initial bonding between cable and duct.

An approximate method of accounting for the effects of friction in post-tensioned prestressing systems has been presented by E. H. Cooley.[11] His method is presented in tabular form for the two examples worked out in this paper.

It consists of splitting up the tensioning unit into a number of sections. Each section is not to exceed 30 ft in length. The ratio of the tensions at the two ends of each section is then calculated. This ratio may be expressed sufficiently accurately as

$$1 - \frac{\mu L}{r}$$

where L and r are the length and radius of the section, respectively, and μ is the friction constant for the prestressing unit employed. A "wobble" factor is included in the work by Cooley but is omitted in this paper.

The "wobble" factor is used to provide for the effects of inaccuracies of alignment of the sheath or duct.

It is sufficiently accurate to assume the length of the section along the member as L. The radius r may be determined approximately, for parabolic sections, in the following manner.

Referring to Fig. 6, it will be seen that

$$y = \frac{4x^2 C_A}{L^2}$$

and

$$\frac{d^2y}{dx^2} = \frac{8C_A}{L^2}$$

Then

$$r = \frac{1}{\frac{d^2y}{dx^2}} = \frac{(0.5L)^2}{2C_A} \quad \text{approximately}$$

also

$$r = \frac{(a_1 L)^2}{2C_1} \quad \text{and} \quad r = \frac{(a_2 L)^2}{2C_2}$$

For example 1 the radius is

$$r = \frac{(70)^2}{2(2.044)} = 1200 \text{ ft}$$

and for example 2 the radii are

$$r = \frac{(35)^2}{2(1.181)} = 519 \text{ ft}$$

$$r = \frac{(24.5)^2}{2(2.036)} = 147 \text{ ft}$$

and

$$r = \frac{(10.5)^2}{2(0.872)} = 63 \text{ ft}$$

A value of $\mu = 0.07$ has been selected to use in illustrating the approximate method suggested by Cooley. The method applied to examples 1 and 2 are given in Tables 1, 2, and 3 and are self-explanatory.

Recommended Procedure

The following procedure may be found useful in starting the design of a prestressed concrete continuous beam.

Using the live and dead loads, compute the maximum negative bending moment at the support and the maximum positive bending moment in the span for the continuous structure as if it were of homogeneous material.

Select a section and estimate the prestressing force. This may be done by multiplying the area of the section by the product of the allowable compressive unit stress and the smaller distance from the centroidal axis to the extreme fiber, then dividing by the depth of the section.

To obtain e below the centroidal axis at the center of the span, divide the maximum live and dead load positive bending moment in the span by the prestressing force.

To obtain e above the centroidal axis at the support, divide the maximum live and dead load bending moment at the support by the prestressing force.

These e values must lie within the beam. Otherwise it is necessary to select a new beam section for trial.

Compute the equivalent prestress loads, if the beam is straight, by use of

$$w_p = -\frac{4\,TC_n}{(a_n L)^2}$$

or, if the beam is haunched, by use of

$$w_p = \left(\frac{e_1 - 2e_2 + e_3}{(\Delta x)^2}\right) T$$

If friction is to be considered, use the average tension over the sectional length for T.

Using the prestress load w_p, analyze the continuous structure.

Using the principle of superposition, add the moments caused by prestress to the moments caused by dead load and live load in order to obtain the values for the curves of maximum moments.

Keeping in mind that there is a normal force caused by prestressing, compute the unit stresses at the critical points.

Check the ultimate load for the beam as if it were a reinforced concrete beam.

Revise design if necessary.

REFERENCES

1. A Study of Continuous Beams and of Some Statically Redundant Systems in Prestressed Concrete by M. Y. Guyon, Circular J No. 8 20th September 1945 of the Institute Technique du Bâtiment des Travaux Public, Translation No. 33 Cement and Concrete Association 1951.

2. Analysis of Continuous Prestressed Concrete Structures by A. L. Parme and G. H. Paris, Proceedings of the First United States Conference on Prestressed Concrete, 1951, p. 195.

3. Designing for Continuity in Prestressed Concrete Structures by A. L. Parme and G. H. Paris, Journal, American Concrete Institute, Sept. 1951, Vol. 23 No. 1 p. 45.

4. Equivalent Load Method for Analyzing Prestressed Concrete Structures by Robert B. B. Moorman, Journal, American Concrete Institute, Jan. 1952, Vol. 23, No. 5 p. 405.

A symposium on prestressed concrete statically indeterminate structures, Sept. 1951, Cement and Concrete Association, 1953:

5. The analysis of prestressed concrete statically indeterminate structures by D. W. Cracknell and W. A. Knight, p. 19.

6. Continuity in prestressed concrete by G. Magnel, p. 77.

7. Determination of continuity bending moments in prestressed continuous beams by E. G. Trimble, p. 95.

8. A theoretical treatment of continuity in prestressed concrete by Y. Guyon, p. 131.

9. Continuous Prestressed Concrete Beams by Fritz Leonhardt, Journal, American Concrete Institute, Mar. 1953 Vol. 24 No. 7 p. 617.

10. The design of prestressed continuous beams of uniform section by R. G. Robertson, Magazine of Concrete Research No. 12, April 1953, p. 97.

11. Friction in post-tensioned prestressing systems by E. H. Cooley, Research Report No. 1, October 1953, Cement and Concrete Association.

12. Aspects Practiques de la Précontrainte par Cables - Le Problème des Frottements by J. Montagnon, Annales de l'Institut Technique du Bâtiment et des Travaux Publics, June 1954 No. 78, p. 503.

Table 1

Tensioning from one end (Example 1)

Section	Length, ft	Radius, r, ft	μ	$\frac{\mu L}{r}$	Section ratio $1-\frac{\mu L}{r}$
AB	17.5	1200	0.07	0.001	0.999
BC	17.5	1200	0.07	0.001	0.999
CD	17.5	1200	0.07	0.001	0.999
DE	17.5	1200	0.07	0.001	0.999
EF	17.5	1200	0.07	0.001	0.999
FG	17.5	1200	0.07	0.001	0.999
GH	17.5	1200	0.07	0.001	0.999
HI	17.5	1200	0.07	0.001	0.999

Point	Tension factor	Stress or tension, kips
A	1.000	610.00
B	0.999	609.39
C	0.998	608.78
D	0.997	608.17
E	0.996	607.56
F	0.995	606.95
G	0.994	606.34
H	0.993	605.73
I	0.992	605.12

Section	Cable extension		Average tension factor	Section length, ft	Effective length, ft
	Jack end	Far end			
AB	1.000	0.999	0.9995	17.5	17.49
BC	0.999	0.998	0.9985	17.5	17.47
CD	0.998	0.997	0.9975	17.5	17.46
DE	0.997	0.996	0.9965	17.5	17.44
EF	0.996	0.995	0.9955	17.5	17.42
FG	0.995	0.994	0.9945	17.5	17.40
GH	0.994	0.993	0.9935	17.5	17.39
HI	0.993	0.992	0.9925	17.5	17.37
					139.44

Table 2

Tensioning from two ends (Example 2)

Section	Length, ft	Radius, r, ft	μ	$\frac{\mu L}{r}$	Section ratio $1-\frac{\mu L}{r}$
AB	17.5	519	0.07	0.002	0.998
BC	17.5	519	0.07	0.002	0.998
CD	24.5	147	0.07	0.012	0.988
DE	10.5	63	0.07	0.012	0.988

Point	Tension factor	Stress or tension, kips
A	1.000	610.00
B	0.998	608.78
C	0.996	607.56
D	0.984	600.24
E	0.972	592.92

Section	Cable extension		Average tension factor	Section length, ft	Effective length, ft
	Jack end	Far end			
AB	1.000	0.998	0.999	17.5	17.48
BC	0.998	0.996	0.997	17.5	17.45
CD	0.996	0.984	0.990	24.5	24.26
DE	0.984	0.972	0.978	10.5	10.27
					69.46
					x2 = 138.92

Table 3

Tensioning from one end (Example 2)

Section	Length, ft	Radius, r, ft	μ	$\frac{\mu L}{r}$	Section ratio $1-\frac{\mu L}{r}$
AB	17.5	519	0.07	0.002	0.998
BC	17.5	519	0.07	0.002	0.998
CD	24.5	147	0.07	0.012	0.988
DE	10.5	63	0.07	0.012	0.988
EF	10.5	63	0.07	0.012	0.988
FG	24.5	147	0.07	0.012	0.988
GH	17.5	519	0.07	0.002	0.998
HI	17.5	519	0.07	0.002	0.998

Point	Tension factor	Stress or tension, kips
A	1.000	610.00
B	0.998	608.78
C	0.996	607.56
D	0.984	600.24
E	0.972	592.92
F	0.960	585.60
G	0.948	578.28
H	0.946	577.06
I	0.944	575.84

Section	Cable extension		Average tension factor	Section length, ft	Effective length, ft
	Jack end	Far end			
AB	1.000	0.998	0.999	17.5	17.48
BC	0.998	0.996	0.997	17.5	17.45
CD	0.996	0.984	0.990	24.5	24.26
DE	0.984	0.972	0.978	10.5	10.27
EF	0.972	0.960	0.966	10.5	10.14
FG	0.960	0.948	0.954	24.5	23.37
GH	0.948	0.946	0.947	17.5	16.57
HI	0.946	0.944	0.945	17.5	16.54
					136.08

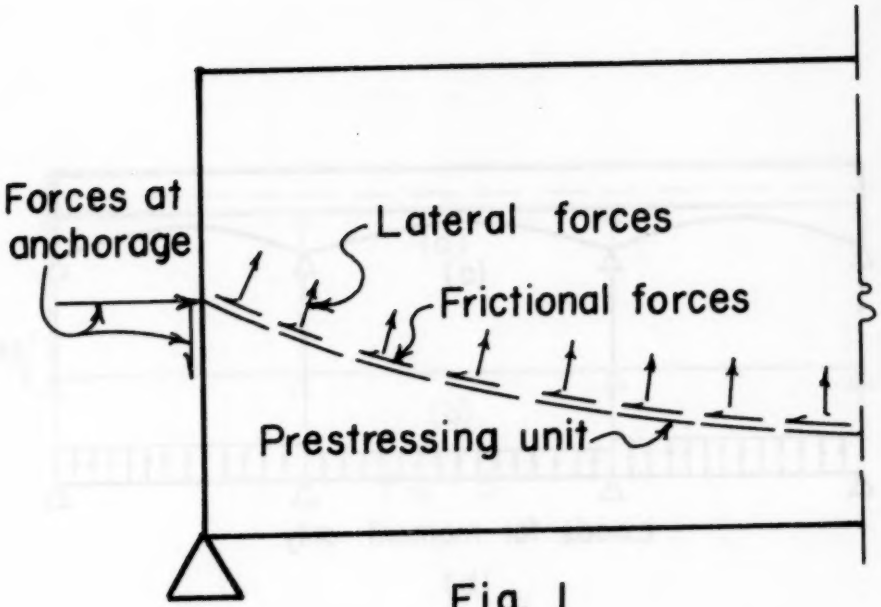
Forces at anchorage
Lateral forces
Frictional forces
Prestressing unit

Fig. 1

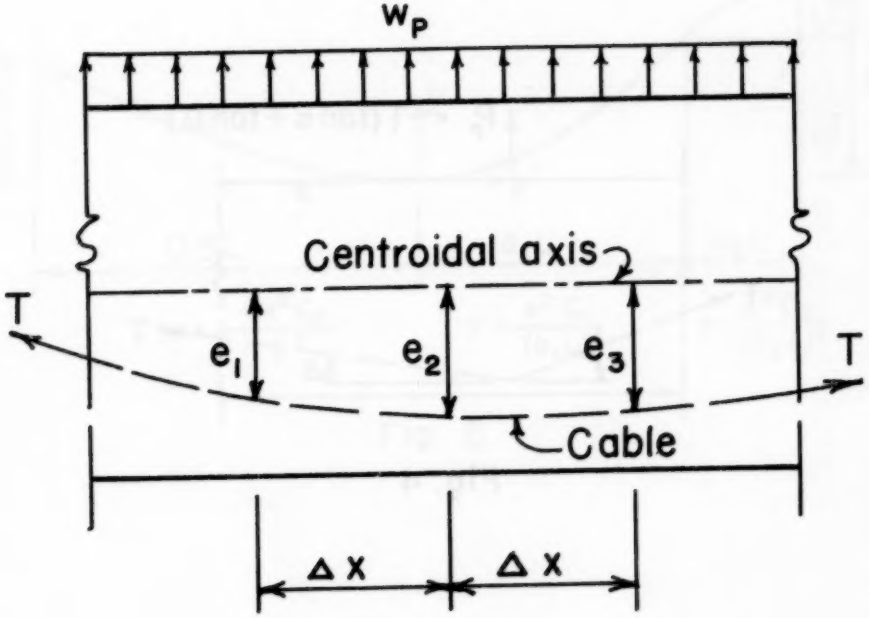
w_p
Centroidal axis
T
e_1
e_2
e_3
T
Cable
Δ x
Δ x

Fig. 2

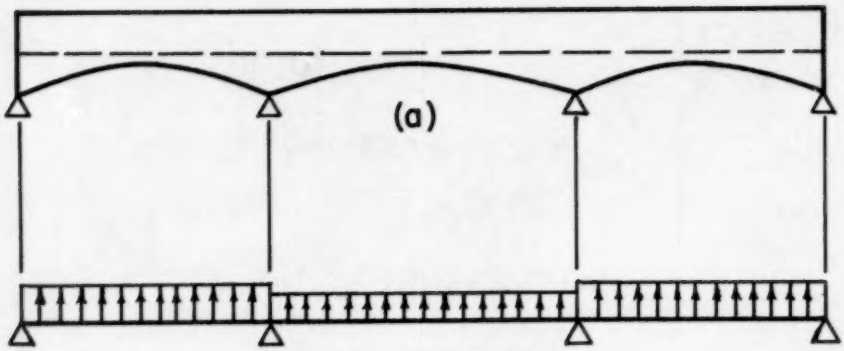

Fig. 3

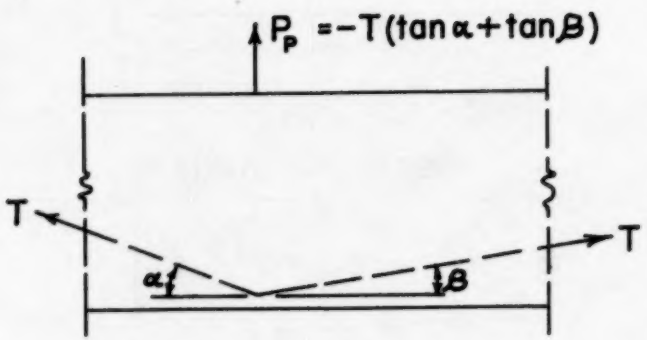

Fig. 4

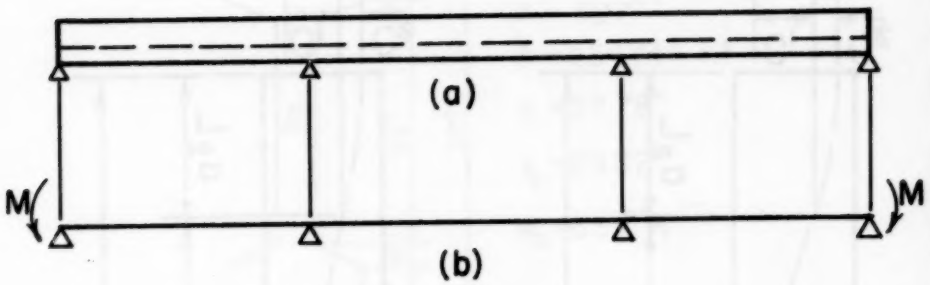
(a)
M
M
(b)

Fig. 5

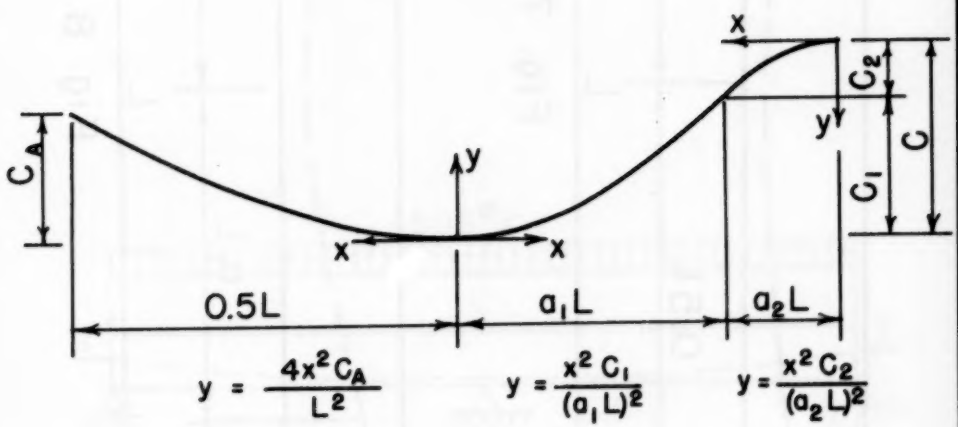
x
C_2
y
C
C_1
C_A
y
x
x
0.5L
a_1L
a_2L
$y = \frac{4x^2 C_A}{L^2}$
$y = \frac{x^2 C_1}{(a_1 L)^2}$
$y = \frac{x^2 C_2}{(a_2 L)^2}$

Fig. 6

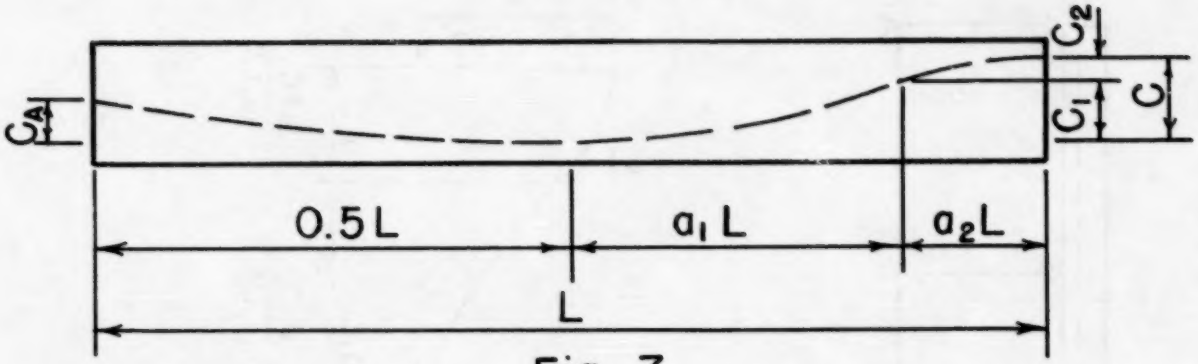

Fig. 7

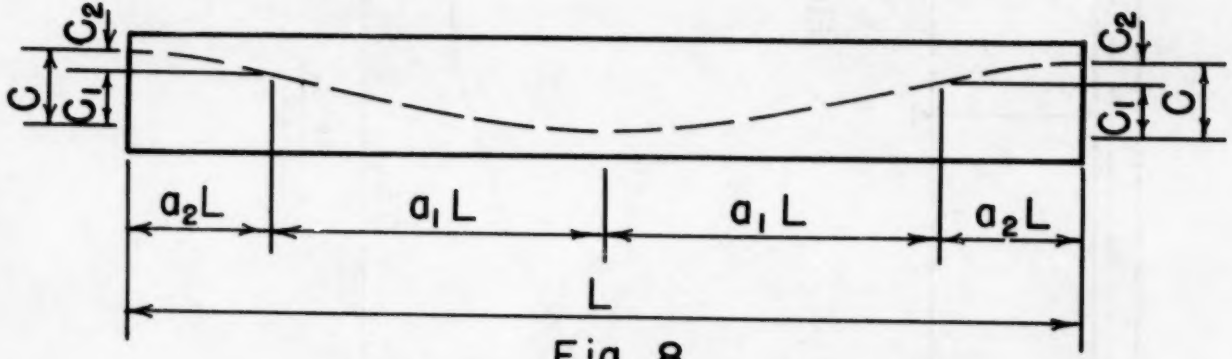

Fig. 8

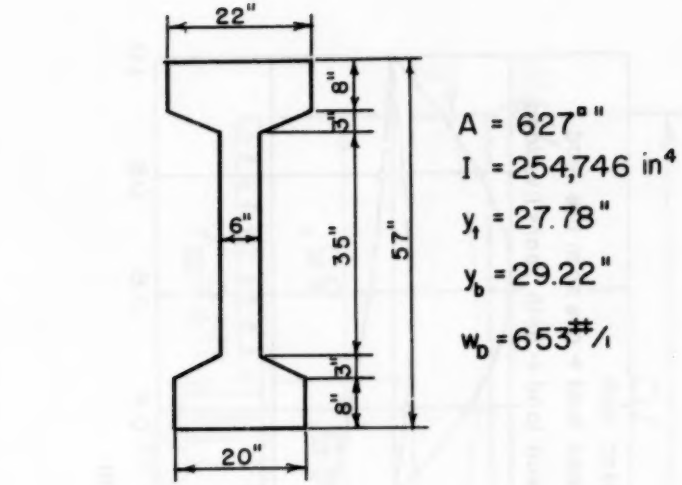
22"
8"
3"
6"
35"
57"
3"
8"
20"
A = 627" "
I = 254,746 in⁴
y_t = 27.78"
y_b = 29.22"
w_D = 653#/'

Fig. 9

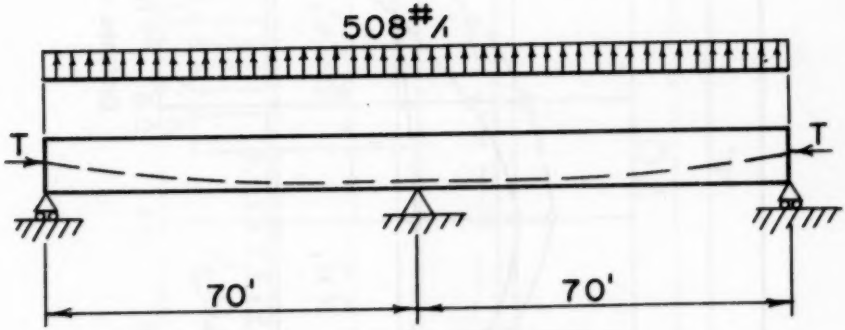
508#/'
T
T
70'
70'

Fig. 10

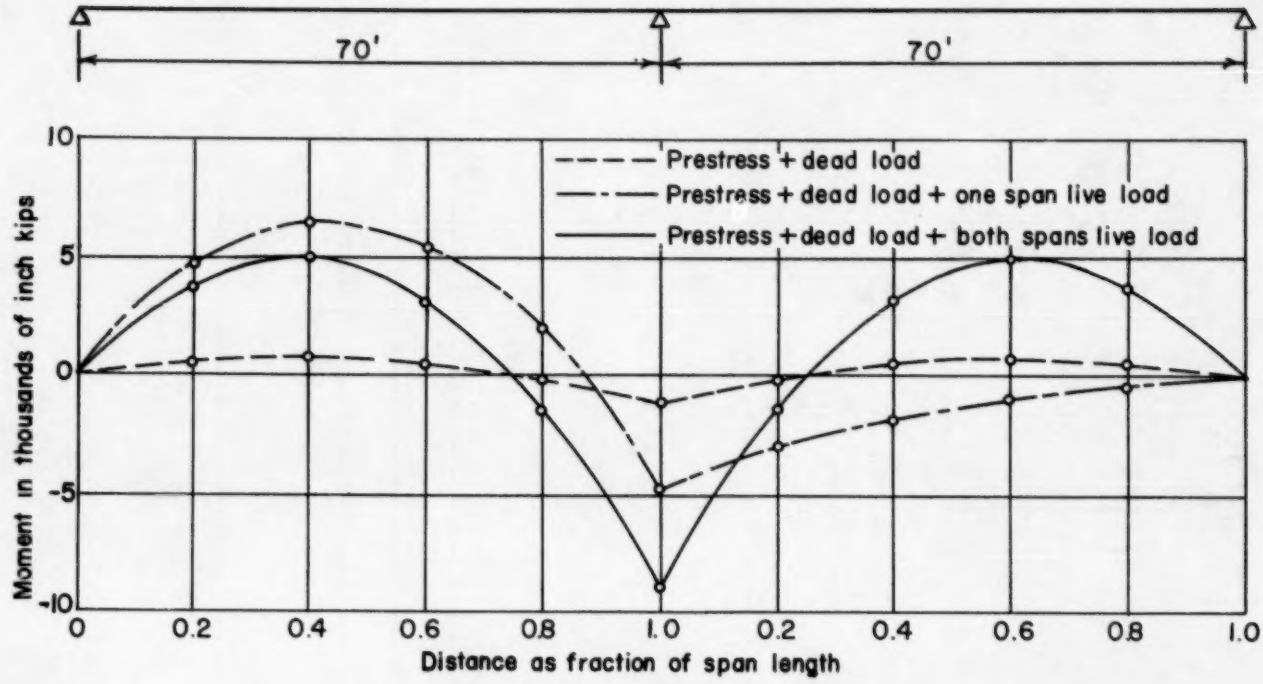
70'
70'
Prestress + dead load
Prestress + dead load + one span live load
Prestress + dead load + both spans live load
Moment in thousands of inch kips
10
5
0
-5
-10
0
0.2
0.4
0.6
0.8
1.0
0.2
0.4
0.6
0.8
1.0
Distance as fraction of span length

Fig. 11

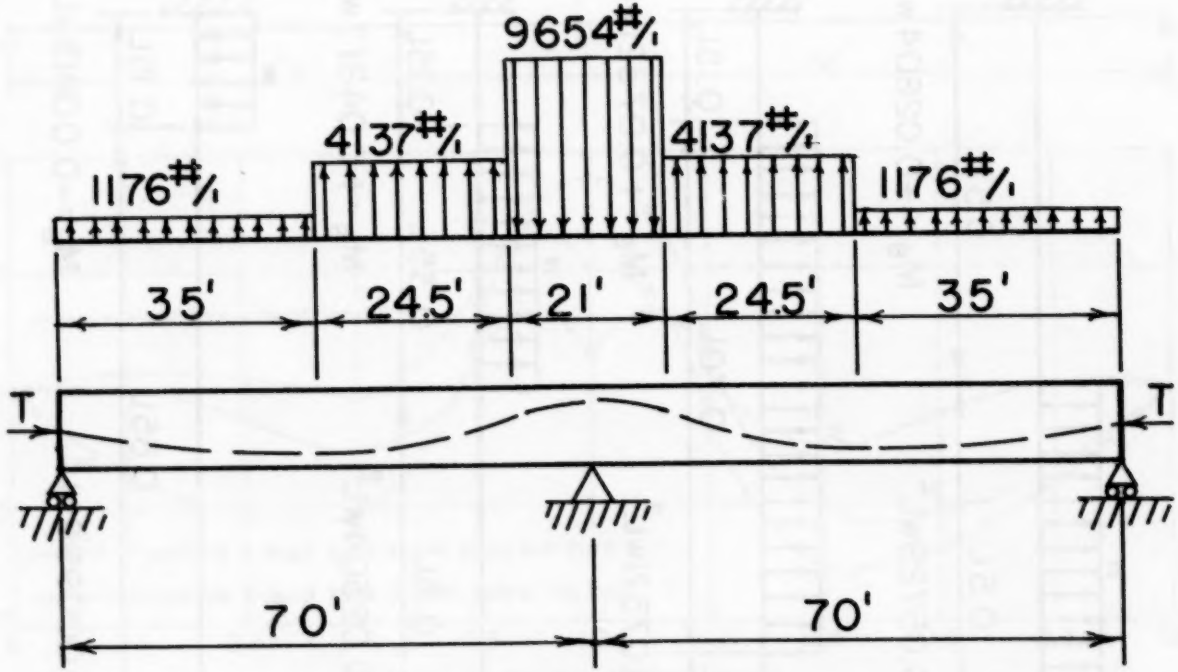

Fig. 12

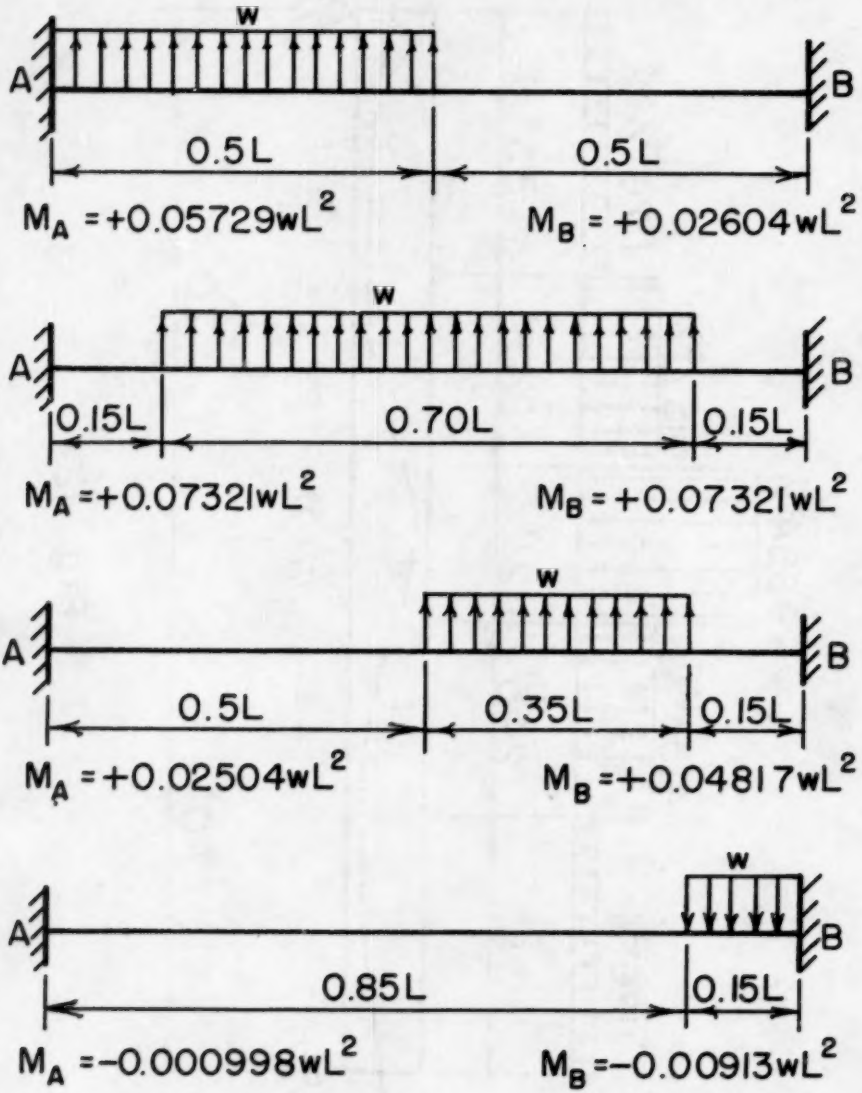
w
A
B
0.5L
0.5L
$M_A = +0.05729wL^2$
$M_B = +0.02604wL^2$
w
A
B
0.15L
0.70L
0.15L
$M_A = +0.07321wL^2$
$M_B = +0.07321wL^2$
w
A
B
0.5L
0.35L
0.15L
$M_A = +0.02504wL^2$
$M_B = +0.04817wL^2$
w
A
B
0.85L
0.15L
$M_A = -0.000998wL^2$
$M_B = -0.00913wL^2$

Fig. 13

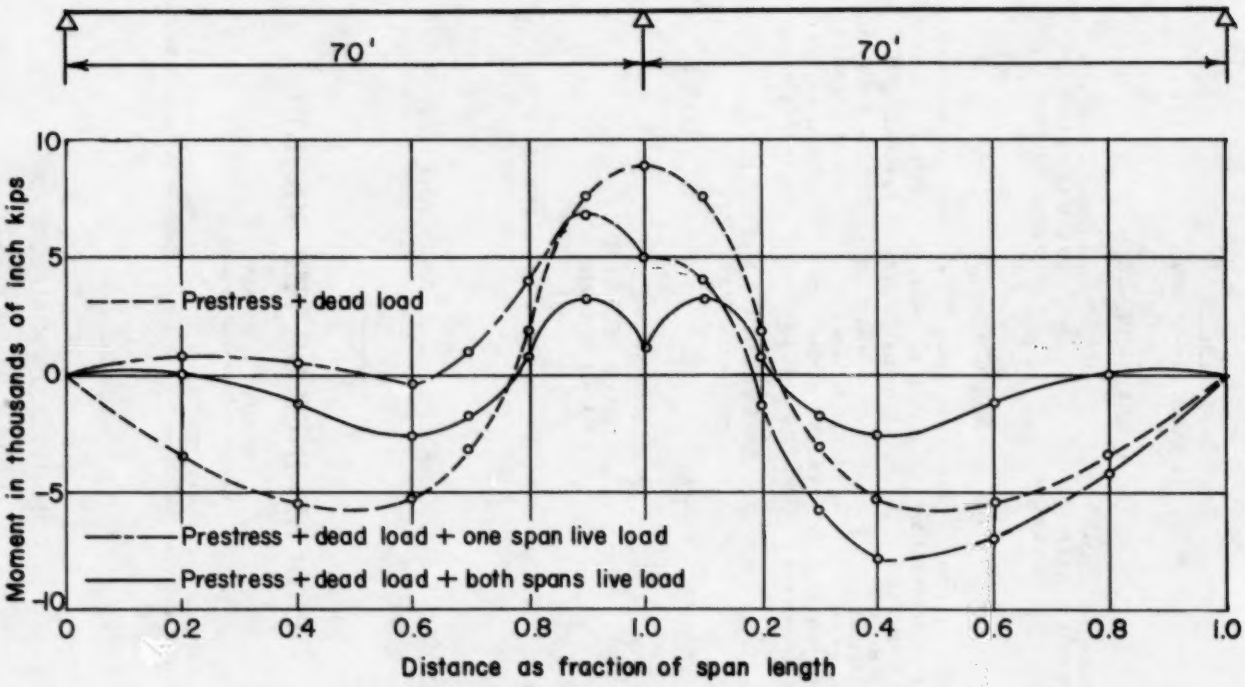
70'
70'
10
5
0
-5
-10
Moment in thousands of inch kips
Prestress + dead load
Prestress + dead load + one span live load
Prestress + dead load + both spans live load
0
0.2
0.4
0.6
0.8
1.0
0.2
0.4
0.6
0.8
1.0
Distance as fraction of span length

Fig. 14

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1955

PRESIDENT

WILLIAM ROY GLIDDEN

VICE-PRESIDENTS

Term expires October, 1955:
ENOCH R. NEEDLES
MASON G. LOCKWOOD

Term expires October, 1956:
FRANK L. WEAVER
LOUIS R. HOWSON

DIRECTORS

Term expires October, 1955:
CHARLES B. MOLINEAUX
MERCEL J. SHELTON
A. A. K. BOOTH
CARL G. PAULSEN
LLOYD D. KNAPP
GLENN W. HOLCOMB
FRANCIS M. DAWSON

Term expires October, 1956:
WILLIAM S. LaLONDE, JR.
OLIVER W. HARTWELL
THOMAS C. SHEDD
SAMUEL B. MORRIS
ERNEST W. CARLTON
RAYMOND F. DAWSON

Term expires October, 1957:
JEWELL M. GARRELTS
FREDERICK H. PAULSON
GEORGE S. RICHARDSON
DON M. CORBETT
GRAHAM P. WILLOUGHBY
LAWRENCE A. ELSENER

PAST-PRESIDENTS

Members of the Board

WALTER L. HUBER
DANIEL V. TERRELL

EXECUTIVE SECRETARY

WILLIAM N. CAREY

ASSISTANT SECRETARY

E. L. CHANDLER

TREASURER

CHARLES E. TROUT

ASSOCIATE SECRETARY

WILLIAM H. WISELY

ASSISTANT TREASURER

CARLTON S. PROCTOR

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

DEFOREST A. MATTESON, JR.
Editor of Technical Publications

PAUL A. PARISI
Assoc. Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

SAMUEL B. MORRIS, *Chairman*

JEWELL M. GARRELTS, *Vice-Chairman*

GLENN W. HOLCOMB
OLIVER W. HARTWELL
ERNEST W. CARLTON
DON M. CORBETT